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DESIGN OF PRESTRESSED TANKS

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SANITARY ENGINEERING DIVISION

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PAPERS

DESIGN OF PRESTRESSED TANKS

By J. M. Crom [1]

Synopsis

In the past, the safe design of circular structures has been extremely difficult, mostly because exact knowledge concerning shrinkage and plastic flow of concrete has been lacking. This paper contains new data for evaluating these phenomena and describes methods that make practicable the use of the high strength of cold-drawn steel wire. Such developments have made it possible to design tanks and other large circular structures on a rational basis, with the assurance that an adequate prestress will be maintained to eliminate cracking of concrete and that important reductions will be made in the weight and in the quantities of critical raw materials required for their construction.

Historical

The first important experimentation on methods of prestressing concrete began in France and Germany during the last two decades of the nineteenth century, principally in connection with the reinforcement of beams and girders. Patents covering these methods were taken out during this period in the United States and in the principal nations of Europe. For many years, however, results were highly discouraging because the prestress in the steel reinforcement was released by shrinkage and plastic flow of the concrete. The early experimenters, C. E. W. Doehring, in Germany (1888), J. G. F. Lund in Norway (1907), M. Koenen,[2] in Germany (1907), E. Freyssinet,[3,4,5,6,7] in France (1928

Note.—Written comments are invited for publication; the last discussion should be submitted by **April 1, 1951.**

[1] Vice-Pres., The Preload Enterprises, Inc., New York, N. Y.

[2] "Wie kann die Anwendung des Eisenbetons in der Eisenbahnverwaltung gefördert werden?" by M. Koenen, *Zentralblatt der Bauverwaltung*, 1907.

[3] "Idées et Voies Nouvelles," by E. Freyssinet, *Science et Industrie*, Construction et Travaux Publics Ed., January, 1933, p. 3.

[4] "Progrès Pratiques des Méthodes de Traitement Mécanique des Bétons," by E. Freyssinet, *ibid.*, Travaux Ed., June 30, 1935, pp. 199–210.

[5] "Une Révolution dans les Techniques du Béton," by E. Freyssinet, Librairie d'Eyrolles et l'Enseignement Technique, Paris, 1936.

[6] "Developments in Concrete Making," by E. Freyssinet, *Concrete and Constructional Engineering*, April, 1936, pp. 209–220.

[7] "Une Révolution dans l'Art de Bâtir les Constructions Précontraintes," by E. Freyssinet, *Science et Industrie*, Travaux Ed., November, 1941.

to 1950), Ewald Hoyer,[8] in Germany (1939), P. H. Jackson, in the United States (1888), J. Mandl,[9] in Austria (1896), G. R. Steiner, in the United States (1908), and others discovered that they would not be able to prestress a structure successfully until they could use concretes and steel of higher elastic values and higher strength values than those available at that time.

Large sums were spent to improve the quality of both materials. Mr. Freyssinet, for instance, produced concrete that hardened rapidly, with compressive values in excess of 14,000 lb per sq in. and used reinforcing steel that had an ultimate strength of about 200,000 lb per sq in. With these new materials he was able to use steel stresses sufficiently high so that the shrinkage and flow of the concrete eliminated only a small percentage of the prestress. As a result, many important structures containing prestressed beams and arches were built in Europe during the years preceding World War II.

In the United States, most of the prestressing work has been done in connection with circular structures such as storage tanks, silos, and large-diameter pipe, although some small precast members such as joists and roof slabs have been built. In the early 1920's, in Minnesota and other locations, A. L. Hewett[10] designed and built a number of water tanks whose walls were prestressed with steel rods of ordinary structural grade. Mr. Hewett and many other engineers believed at that time that shrinkage and plastic flow of concrete were almost negligible, and that it was not necessary to consider these factors seriously in tank design.

Although many successful tanks were built under this assumption, it gradually became apparent that there were defects either in the designs or in the methods of construction. Occasionally there were serious cracking and leaking which could not be explained and costly repairs had to be made. Pneumatic mortar or gunite, to use the more common term, was used for the walls and domes of some of these early tanks, since it had greater density and imperviousness than ordinary concrete and because its use practically eliminated construction joints. About 1941, however, it was rumored that the shrinkage rate of gunite was several times that of conventional concrete. These rumors could not be refuted, because of the lack of authentic data.

Accordingly, an interested commercial organization, in collaboration with the Massachusetts Institute of Technology (in Cambridge) and other institutions, sponsored a series of tests to determine not only the comparative shrinkage rates of concrete and gunite, but also the characteristics of these materials with respect to shrinkage, plastic flow, and other factors that might affect prestressing operations.

As the results of these tests became available, it was apparent that shrinkage and flow were principally responsible for the failures that had occurred. It was also obvious that new methods for maintaining stresses in steel would have to be devised if prestressed tank structures were to remain consistently stable under normal operating conditions.

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[8] "Der Stalsaitenbeton," by Ewald Hoyer, *Verlagsgesellschaft*, Otto Elsner, Berlin, 1939.

[9] "Zur Theorie der Cement-Eisen-Constructionen," by J. Mandl, *Zeitschrift*, Oesterreichischer Ingenieur-und Architekten-Verein, November 6, 1896, pp. 593–596; November 13, 1896, pp. 605–609.

[10] "Water Tanks of Reinforced Concrete," by A. L. Hewett, *Journal*, A.W.W.A., Vol. 28, 1936, p. 43.

Shrinkage and Plastic Flow

Concrete.—Various testing laboratories, and writers in the technical press,[11] have repeatedly demonstrated that the shrinkage of concrete is due principally to drying out or dehydration. Curing has little effect on the ultimate amount of shrinkage. Franklin R. McMillan has shown that concrete, if kept saturated with water, will not shrink, but shrinkage will progress rapidly as soon as concrete begins to dry.[12] Resaturation will cause concrete to expand, but will not return it to its original dimensions when shrinkage has been under way for any appreciable length of time. Concretes with low water-cement ratios shrink less than concretes with higher water-cement ratios. Concrete that has been dried in a baking oven is subjected to maximum shrinkage, having a coefficient of 0.0010 and more.

Tests reported in 1946 show[13] that, over a 5-year period, concrete under a constant load approximating 500 lb per sq in. may attain a deformation defined by a coefficient of 0.0006. Thus maximum shrinkage and plastic flow may express a total coefficient of 0.0016, which approaches a release in steel stress of 48,000 lb per sq in., with E_s at 30,000,000 lb per sq in.

Mr. Freyssinet[3,4,5,6,7] has cited conditions under which the combined losses from plastic flow and shrinkage in both steel and concrete have reached totals equivalent to a coefficient of 0.0024, or of 72,000 lb per sq in. in steel. The data in this paper, however, are based on the use of steels and concretes of high unit values; and therefore the coefficients used for the computations reported herein are considered conservative.

Gunite or Pneumatic Mortar.—Loaded and unloaded specimens of concrete and pneumatic mortar were under observation in the laboratories of the Massachusetts Institute of Technology for more than 1,600 days. The results at the end of 400 days were published in 1946.[14] These tests demonstrated that: (*a*) The concrete specimens shrank about one third more than the pneumatic mortar specimens; and (*b*) the extent of plastic flow in the concrete specimens was greater than that in the pneumatic mortar specimens.

Reinforced Concrete and Pneumatic Mortar Under Stress.—The most significant tests made, however, were those designed to measure the loss of steel stress in reinforced concrete and pneumatic mortar under conditions that paralleled as closely as possible those that would occur in the walls of an empty tank in the field.

The specimens for the tests were 4-in. by 4-in. bars of concrete and pneumatic mortar, 24 in. long. The mortar mix was 1:3 and the concrete mix was 1:2.1:3.2. Reinforcing bars were $\frac{3}{8}$-in. and $\frac{1}{2}$-in. round rods, intermediate grade. Each specimen was made under standard testing laboratory conditions except that the pneumatic mortar bars were shot in the field by operators with experience in handling this material.

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[11] "Effect of Plastic Flow and Volume Changes on Design," by C. T. Morris, Rept. of Committee 313, *Journal*, A.C.I., November–December, 1936, pp. 123–128.

[12] "Shrinkage and Time Effects in Reinforced Concrete," by Franklin R. McMillan, *Bulletin No. 3*, Studies in Engineering, Univ. of Minnesota, Minneapolis, Minn., p. 29, Fig. 13.

[13] "Theory of Plates and Shells," by S. Timoshenko, McGraw-Hill Book Co., Inc., New York, N. Y., 1st Ed., 1940.

[14] "Shrinkage and Plastic Flow of Pre-Stressed Concrete," by Howard R. Staley and Dean Peabody, Jr., *Journal*, A.C.I., January, 1946, p. 232, Fig. 2.

The specimens were loaded by using external plates and threaded rods so as to simulate, as nearly as possible, the prestressing operations then in use. The sectional areas of the rods were adjusted to provide high, intermediate, and low loadings for both the concrete and the pneumatic mortar specimens. These values (in pounds per square inch) were, respectively, 2,390, 1,550, and 920 to 935. Readings were also taken on parallel sets of unloaded bars to

TABLE 1.—STUDY OF STRESS LOSSES RESULTING FROM SHRINKAGE AND PLASTIC FLOW

Item	Description	Staley-Peabody Tests[a]				As Assumed for High-Strength Steel			
		Steel Stresses,[b] f_s		Concrete Stresses, f_c		Steel Stresses,[b] f_s		Concrete Stresses, f_c	
		Gunite	Concrete	Gunite	Concrete	Gunite	Concrete	Gunite	Concrete
(1)	(2)	(3)	(4)	(5)	(6)	(3)	(4)	(5)	(6)
	(a) High Load[c]								
	Areas, in Square Inches:								
1	Concrete section, A_c	17	16	17	16	17	16	17	16
2	Steel section, A_s	1.18	1.18	1.18	1.18	0.271	0.255	0.271	0.255
	Stress, in Pounds per Square Inch:								
3	Initial unit stress	34,330	32,515	2,390	2,390	150,000	150,000	2,390	2,390
4	Stress at 400 days	13,700	8,150	950	600	129,370	125,650	2,060	2,000
	Loss of Stress (Line 3 − Line 4):								
5	In pounds per square inch	20,630	24,355	1,440	1,790	20,630	24,350	330	390
6	Percentage (line 5/line 3)	60	75	60	75	13.7	16.2	13.7	16.2
	(b) Low Load[c]								
	Area, in Square Inches:								
7	Concrete section, A_c	17	16	17	16	17	16	17	16
8	Steel section, A_s	0.44	0.44	0.44	0.44	0.104	0.100	0.104	0.100
	Stress, in Pounds per Square Inch:								
9	Initial unit stress	35,545	34,000	920	935	150,000	150,000	920	935
10	Stress at 400 days	15,100	10,000	390	275	129,555	126,000	795	780
	Loss of Stress (Line 9 − Line 10):								
11	In pounds per square inch	20,445	24,000	530	660	20,445	24,000	125	150
12	Percentage (line 11/line 9)	57	71	57	71	13.6	16.0	13.6	16.0

[a] "Shrinkage and Plastic Flow of Pre-Stressed Concrete," by H. R. Staley and Dean Peabody, Jr., *Journal*, A.C.I., January, 1946, p. 229. [b] Unit Stress in steel, $f_s = \frac{f_c A_c}{A_s}$. [c] Computations for medium loads have been omitted to conserve space.

determine the deformation that could be attributed only to shrinkage, and thus, by deduction, to measure the amount of plastic flow.

Table 1 gives the stress losses in the loaded bars at the end of 400 days as reported by Messrs. Staley and Peabody. These losses were the principal data which the tests were designed to determine. The losses in steel stress shown in Cols. 3 and 4 were caused by the shortening of the concrete and pneumatic mortar sections as the result of shrinkage and plastic flow. The bars were

shortened by approximately the same amount irrespective of the load that had been imposed on the steel. The variation in the length of the respective bars was used to measure the losses of stress in the rods.

Before the results of these tests could be applied to the construction of tank walls in the field, it was necessary to make allowance for a number of additional factors which experience has shown also may be responsible for losses of stress.

The Staley-Peabody tests in Table 1, for instance, do not include losses incurred during the beginning of the tests, because the rods were retightened at the end of the second day. The amount of loss was determined by examining the unloaded specimens. There must be included, therefore, an additional loss of about 3,000 lb per sq in. due to initial shrinkage (as determined by examining the unloaded specimens). A loss of this magnitude would certainly take place if concrete was kept saturated with water to the time when it was stressed.

In addition, the tests were made under a constant temperature of 70° F and a humidity of 50%. Under some field conditions the humidity is considerably lower, and an additional 4,000 lb per sq in. should be included to take care of this contingency.

During the supervision of numerous rod-stressing operations in the field, the writer has repeatedly noted variations in stress as high as 10,000 lb per sq in. Lever arms, rusted threads, temperature changes, sun heat, inaccurate stressometer measurements, and other factors can easily account for an error of this magnitude.

Thus, the maximum losses, in pounds per square inch, that might be expected when using band rods and turnbuckles are as follows:

Description	Pneumatic mortar	Concrete
Shrinkage and flow (from Table 1)	21,000	24,000
Initial shrinkage loss	3,000	3,000
Allowance for low humidity, from laboratory oven tests and field tests	4,000	4,000
Possible variation in rod stressing	10,000	10,000
Total	38,000	41,000

With losses of this magnitude, it can readily be seen that, under the low initial prestresses, the tank walls reverted rapidly to an unprestressed state. Serious cracking and trouble due to tension in the concrete should not therefore be surprising. The designers had not contemplated deformations of concrete so great as those which actually occurred.

Little or nothing could be done to reduce appreciably the shrinkage and flow of the concrete. In these respects the poorer concretes do not differ very much from the better ones. There is a remedy. The use of higher strength steels with much higher prestresses will for practical purposes completely counteract losses of stress due to these deformations of the concrete.

To illustrate, there has been included in Table 1 a second set of values based on the use of the high-strength steel. The steel stresses are assumed to have been increased with relative reductions in steel areas. The loadings on the

concrete are left as before and the shrinkage and flow are assumed to be the same. The amount of loss in unit stress of the reinforcing will be the same as before but the percentage loss is vastly different and this is the interesting feature. The unit stresses lost in both concrete and steel are now about 16% or less whereas with the use of the structural grade steel and comparatively low stresses they averaged around 67%.

Furthermore, the use of the high-strength steel permits an overstress to take care of the small percentage loss so that the designed prestress is always within the limits intended. For instance, if the reinforcement available is strong enough to take an initial stress of 140,000 lb per sq in. and losses of 35,000 lb per sq in. in the steel are possible, although improbable, then a working stress of 105,000 lb per sq in. (140,000 lb per sq in. − 35,000 lb per sq in.) is practical when it can be produced in the field. Under such conditions there is always a residual compressive stress in the concrete. Tension of concrete results merely in the release of compression.

Circumferential Prestressing of Tank Walls

To obtain the initial prestress of approximately 140,000 lb per sq in. required for such a design, it was first necessary to find a reinforcing material of high strength and elasticity. Cold-drawn, high-carbon wire possesses these characteristics. The ultimate strength of this wire varies from 200,000 lb per sq in. to 240,000 lb per sq in., and it has a yield point of about 85% (0.2% by the offset method) of the ultimate. There remained the problem of applying the wire, under the desired stress, around the walls of circular structures. This was accomplished through the development of a traction machine that would simultaneously place and stress the wire to the desired specifications without any friction between the wire and the concrete. It thus became practicable to design and build prestressed reinforced-concrete tanks that would be stable and free from cracking and leaking. The largest circular concrete tanks in existence now contain wire reinforcement stressed by these machines.

Where several layers of the wire are used, the inner layers act with the supporting concrete, and, in effect, take some compression. There is little loss of stress on this account since, under live load, the inner wires will expand with the concrete and act in unison with the outer wires to take their full part of the load. The load resistance of the steel ($R = f_s A_s$) will be in accordance with its total sectional area (A_s) and the average stress is (f_s) $= R/A_s$, as would be the case with a cable composed of multiple strands.

The use of the wire also has the advantage that its modulus of elasticity averages approximately 26,000,000 lb per sq in. against 30,000,000 lb per sq in. for steel rods. Therefore, the wire will stretch further (in inches per pound of stress) than rods, within the elastic range of the material. Since plastic deformation of the supporting concrete thus eliminates a smaller percentage of the stretch, the losses in stress when wire is used are less than those when rods are used.

In addition, the machine-controlled wire-stressing operation is highly accurate. Where threaded rods are used, variation in stress, due to friction between the rod and the concrete surface, may amount to 25% when the rods are stressed

to 40,000 lb per sq in. Attempts have been made in Europe to stress wires around a circular wall by jacking the wire at one point; but it was found that, unless elaborate means were employed to reduce this friction, it was impossible to obtain sufficient stress in the wire on the side opposite the stressing point to make the structure dependable. In the case of the machine-controlled wire stressing, where the wire is already stressed before it comes in contact with the wall, variations in stress are less than 3% of the initial stress. For this reason, 35,000 lb per sq in. is an adequate allowance for stress losses with machine-controlled wire stressing; but at least 40,000 lb per sq in. must be allowed for prestressed rods with turnbuckles, and more than 70,000 lb per sq in. if wire is used which is stressed after it is in contact with the surface, depending on the friction coefficient and methods used to overcome it.

Comparative Tank Designs

It may be of interest to compare the band stresses in the walls of a concrete water tank under designs using high-carbon wire, prestressed rods, and un-

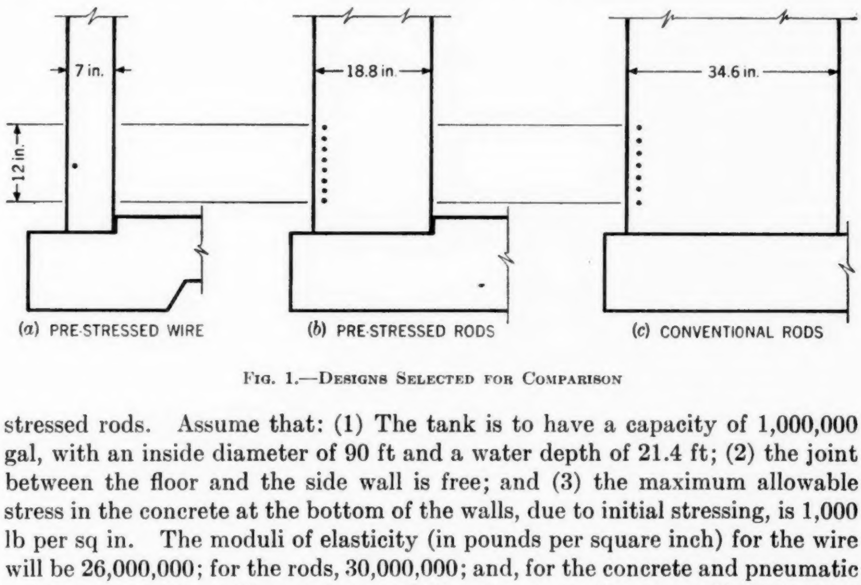

Fig. 1.—Designs Selected for Comparison

stressed rods. Assume that: (1) The tank is to have a capacity of 1,000,000 gal, with an inside diameter of 90 ft and a water depth of 21.4 ft; (2) the joint between the floor and the side wall is free; and (3) the maximum allowable stress in the concrete at the bottom of the walls, due to initial stressing, is 1,000 lb per sq in. The moduli of elasticity (in pounds per square inch) for the wire will be 26,000,000; for the rods, 30,000,000; and, for the concrete and pneumatic mortar, 3,000,000. The minimum yield point stress for wire will be 185,000 lb per sq in. and for the rods it will be 60,000 lb per sq in. (In the past, many rod prestressed tanks were designed using an initial stress of approximately 28,000 lb per sq in. and a working stress of about 25,000 lb per sq in. Such stresses proved insufficient. By making use of the highest practicable initial stresses, and low working stresses, it is possible to design a fairly safe rod tank such as that shown in Col. 2, Table 2. Under extreme conditions, however, circumferential tension due to plastic deformation and temperature changes may be sufficient to cause the walls of this tank to crack.)

To facilitate comparison, the designs for these three types of tanks are presented in Table 2. Figs. 1(*a*), 1(*b*), and 1(*c*) show the thicknesses of the re-

spective shells in lines 1, 2, and 3, Table 2. The condition for design is based on a tensile stress not to exceed 300 lb per sq in. for 3,000-lb concrete.[15] In line 3, Col. 20, the maximum stress is 299 lb per sq in.

The design of the tank in line 3 is based on a free sliding base, and is thus comparable to the tank designs in lines 1 and 2. If a fixed or hinged base had been used, the wall thickness would have less because the cantilever action between the floor and the walls would have absorbed some of the band tension over the bottom of the wall. The Portland Cement Association,[15] however, uses the coefficient of shrinkage of 0.0003, whereas for these computations,

TABLE 2.—COMPARISON OF DESIGN METHODS (STORAGE TANK: INSIDE

Line	Design (see Fig. 1)	DESIGN CRITERIA				DESIGN DIMENSIONS					
		f_s, in lb per sq in.	f_{si}, in lb per sq in.	f_{ci}, in lb per sq in.	$n = \frac{E_s}{E_c}$	A_s	A_c (prestress)	$t = \frac{A_c}{12}$	Conventional		$P = \frac{A_s}{A_c}$
									t	A_c	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	Prestressed wire.....	−100,000	−140,000	+1,000	8.7	0.6	84	7	...	...	0.007
2	Prestressed rods.....	− 12,000	− 45,000	+1,000	10	5.0	225	18.8	...	...	0.022
3	Conventional rods...	− 14,000	....	− 300	10	4.28	...	...	34.6	415	0.0103

[a] Definition of column headings:

Column	Definition
1	The cross sections corresponding to lines 1, 2, 3, are shown in Figs. 1(*a*), 1(*b*), and 1(*c*), respectively. The formula for the design of line 3 (Fig. 1(*c*)) was based partly on equations presented in "Circular Concrete Tanks Without Prestressing," *Bulletin No. ST–57*, Structural Bureau, Portland Cement Assn., Chicago, Ill., October, 1947.
3	f_s = unit working stress used in the design.
4	f_{si} = maximum allowable initial unit steel stress.
5	f_{ci} = maximum allowable initial unit concrete stress.
7	$A_s = \frac{\gamma h r}{f_s} = \frac{62.5 \times 21.4 \times 45}{f_s} = \frac{60{,}000}{f_s}$, in square inches.
10	t = thickness of wall, in inches $= \frac{C E_s + f_s - n f_c}{12 f_c f_s} \times \gamma h r$ (in which $C = 0.0006$).
11	$A_c = 12\, t$ in the conventional tank (see Fig. 1(*c*)).
13	f_{sL} = stress in steel (with tank empty), after a loss of 35,000 lb per sq in. for line 1; and 40,000 lb per sq in. for line 2.
14	f_{cL} = stress in concrete $= \frac{f_{sL} A_s}{A_c}$ (with tank empty), after a loss of 35,000 lb per sq in. for line 1, and 40,000 lb per sq in. for line 2.

based on field observations and laboratory tests, the coefficient of shrinkage was set at 0.0006. In any large tank, it would seem ,highly dangerous to use less than the full amount of band steel required by the hydrostatic load. Therefore no consideration has been given to possible assistance due to cantilever action between base and wall.

The most significant data in Table 2 are contained in Col. 20, which sets down the band stresses of the full tanks after anticipated losses have been taken into account. It will be noted that concrete of the wire-wound tank still retains 78 lb per sq in. of circumferential compression. The data in Cols. 15 and 16 agree with data reported by Torata Matsumoto in 1921.[16]

[15] "Circular Concrete Tanks Without Prestressing," *Bulletin No. ST-57*, Structural Bureau, Portland Cement Assn., Chicago, Ill., October, 1947, p. 3.

[16] "A Study of the Effect of Moisture Content Upon the Expansion and Contraction of Plain and Reinforced Concrete," by Torata Matsumoto, *Bulletin No. 126*, Univ. of Illinois Eng. Experiment Station, Urbana, Ill., p. 27, Fig. 12.

Cols. 22 and 23 are also interesting, since they show what would happen if general cracking in a vertical direction should take place. With the unstressed tank, leakage due to an increase in the circumference of the walls would be substantial. With the prestressed rod tank, insurance against vertical cracking would be partly dependent on the tensile values of the concrete. A vertical crack in the wire-wound tank would not leak; and the steel unit stress would not be increased on this account. This has been proved repeatedly by actual conditions in the field. Before being banded with wire, shrinkage cracks, as wide as ¼ in., have frequently been observed in tank walls. After the prestressing had been completed, these cracks were not only leakproof, but they could be detected only under close scrutiny.

DIAMETER, 90.0 FT; HEIGHT, 21.4 FT; PLUS INDICATES COMPRESSION)[a]

ANALYSIS AFTER ASSUMED LOSSES FOR PLASTIC DEFORMATION								VERTICAL CRACKING[b]			QUANTITIES[c]		Line
f_{sL}	f_{cL}	f_{ss}	f_{cs}	$f_c\Delta$	$f_s\Delta$	f'_s	f'_c	$f_s\Delta$	f_{sc}	Col. 22 / E_s	Concrete (cu yd)	Reinforcement (lb)	
In Pounds per Square Inch													
(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(1)
−105,000	+750	....	....	−672	−5,820	−110,820	+ 78	−5,820	0	0	145	6,800	1
− 5,000	+110	....	....	−218	−2,180	− 7,180	−107	−2,180	− 4,820	0.54	395	58,500	2
....	....	+16,300	−167	−132	−1,320	+ 14,980	−299	−1,320	−28,980	3.25	660	51,200	3

Column	Definition
15	f_{ss} = unstressed compression rods $= \dfrac{C\,E_s}{1 + p\,n}$, in which $C = 0.0006$.
16	f_{cs} = unstressed tension rods $= \dfrac{C\,E_s}{1 + p\,n}$, as in "Circular Concrete Tanks Without Prestressing," *Bulletin ST-57*, Structural Bureau, Portland Cement Assn., Chicago, Ill., October, 1947, p. 42.
17	$f_c\Delta$ = concrete stress caused by filling the tank $= \dfrac{\gamma\,h\,r}{A_c + n\,A_s}$.
18	$f_s\Delta$ = steel stress caused by filling the tank $= n\,f_c\Delta$.
19	f'_s = Col. 13 (or Col. 15) plus Col. 18.
20	f'_c = Col. 14 (or Col. 16) plus Col. 17 (full tank).
21	$f_s\Delta$ = change in steel stress caused by filling the tank.
22	f_{sc} = change in steel stress in the case of cracks from any cause (Cols. 3 to 19).
23	E_s = maximum increase in circumference of tank $= 90 \times 12 \times \pi \times \dfrac{\text{Col. 22}}{E_s}$.
24	= volume of concrete in the straight wall, not footings.

[b] Stress analysis for the case of vertical cracking. [c] Band reinforcement and concrete.

It is difficult to fix a factor of safety for a tank structure because, almost invariably, overflow pipes limit the head to that of the design. Should the tank be covered with a dome, it is usual practice to place a hatch near the dome ring which will be pushed open by the liquid if the overflow pipe should become stopped up; therefore, the principal factor desired is safety against cracking. In the case of prestressed wire walls there is a real factor of safety in this respect. With unstressed tanks, this factor does not apply because, by virtue of the foregoing calculations, they must crack unless the tensile strength of the concrete is sufficient to carry all loads. In other words, the concrete walls of the conventionally designed tank must carry not only the hydrostatic load but also the expansive stress of the rods, which become compressed by the shrinkage

of the concrete to about 16,300 lb per sq in. (Col. 15, Table 2). This condition is illogical and unsafe.

Vertical Reinforcement

In designing the reinforcement for the walls of a prestressed tank, it must be kept in mind that the conditions are different from those for the reinforcement of a conventional tank. With the latter, aside from shrinkage, the principal factor is the radial force exerted by the liquid, and its tendency to bend the walls outwardly.

In prestressed tank design, however, the bending forces are reversed. If shrinkage and plastic flow did not have to be taken into account, the radial inward forces of the prestressed reinforcement would be designed to counterbalance exactly the radial outward forces of the liquid load. Obviously, when the tank is empty, the radial forces exerted by the prestressed reinforcement are unopposed except by the concrete. Consequently, moments are developed in the walls, and vertical reinforcement must be provided to take care of them.

The computations of such moments are complex. In effect, a circular wall is a cylinder or a deep ring. It is not correct, therefore, to consider a unit vertical section of the wall and assume that it is acted upon as a simple beam supported at the ends. Circumferential ring stresses, as well as the moments due to bottom and top connections, must also be taken into consideration.

S. Timoshenko[13] and others have analyzed these conditions. Excellent tables for use in the computation of such moments, based on Mr. Timoshenko's work, are available.[15] The steel sectional area in the tension faces of the tank wall, required to take the stresses as computed from the moment tables, is determined as follows:[15]

$$A_s = \frac{12\,M}{f_s\,j\,d} \qquad (1)$$

in which d is the effective depth of beam and j is the ratio of the distance between the resultants of the compressive and tensile stresses to the effective depth. However, in using the tables, it must always be remembered that the tension faces of the walls of a prestressed tank are exactly opposite to those of a conventionally designed reinforced tank, because the stressing of the horizontal reinforcement causes the walls to bend inward. When the tank is filled, the liquid load will serve only to reduce this inward bending.

In addition, the tables are predicated on the use of hinged and fixed walls. Thus they do not quite fit the conditions assumed for the designs in Table 2, which, for purposes of comparison, were based on the use of a free sliding joint. This joint, however, cannot be considered free in the sense that it slides easily since, even with good lubrication such as asphalt or greased paper, it will not (except by special provision) move to a position at which all lateral restraint is eliminated.

New computations were therefore made on the assumption that a free sliding joint has a friction coefficient of 0.5. This ratio, based on the writer's observations during many field construction operations, seems sufficiently accurate for practical purposes and is on the side of safety. Moment curves based on the use of this kind of joint and applicable to the wire-wound tank

(Fig. 1(a)) are shown in Fig. 2. The maximum moment in the side walls resulting from the stressing of the band reinforcement is approximately 2,850 ft-lb and it exists about 3.8 ft above the floor.

Using Eq. 1, the steel required for the inside face of the wall will be: $A_s = \frac{12\,M}{f_s\,j\,d} = \frac{12 \times 2{,}850}{20{,}000 \times 0.88 \times 5.5} = 0.35$ sq in.—which ordinarily indicates the use of nonprestressed $\frac{5}{8}$-in. round rods on 10-in. centers. This type of reinforcement is illogical. It would resist shrinkage of the concrete and set up adverse stresses in both steel and concrete, as follows: $f_{cs} = \frac{E_c\,C \times p\,n}{1 + p\,n}$ $= \frac{3{,}000{,}000 \times 0.0007 \times 0.0042 \times 10}{1 + 0.042} = 85$ lb per sq in. The comparative stress in the steel resisting this tension in the concrete is, theoretically: $\frac{A_c\,f_{cs}}{A_s}$ $= \frac{84 \times 85}{0.35} = +\,20{,}000$ lb per sq in.

Bond failure might reduce this stress to some extent; but, in any event, the steel could not resist any part of the moment until after the concrete had ruptured and the section had elongated sufficiently for the rods to act in tension. Furthermore, the prestressing imposes a continuous load on the concrete of the walls. When the tank is filled, the circumferential stresses will be partly released by the liquid load, thus reducing the effects of plastic flow. There will be no release vertically. Therefore, to maintain actual resistance to wall moments in large tanks, it is necessary to include a large amount of prestress in the vertical reinforcement.

Many designers have believed that no useful purpose would be served by prestressing tank walls in a vertical direction. The writer has inspected more than 100 conventionally designed reinforced concrete tanks and silos, and has found horizontal cracking in the walls almost invariably. Such cracks not only disfigure the structure, but also result in leakage which may cause early loss of the tank due to freezing and thawing of water in the fracture planes. The writer's experience with tanks that have been prestressed circumferentially also strongly indicates the need for vertical prestressing. Since about 1942 approximately 50 tanks have been built under designs using vertical prestressed steel as described herein. Results have been most satisfactory and no repairs have been necessary.

The prestressing of the vertical reinforcement has been difficult and expensive. Since 1942 several methods have been employed with varying degrees of success, but it is only recently that fully satisfactory results have been achieved. For small tanks whose side walls are less than 15 ft high, conventional reinforcement has sometimes been used. A number of larger tanks has been built with prestressed rods as the vertical reinforcement. Although this method was expensive, the results were sometimes satisfactory when the rods were stressed as much as possible and then bonded into the concrete. The use of slip rods, that is, rods covered with a plastic, was very unsatisfactory. This is illustrated by the history of a 2,000,000-gal tank built during World War II for the United States Navy Department at its Great Lakes Training

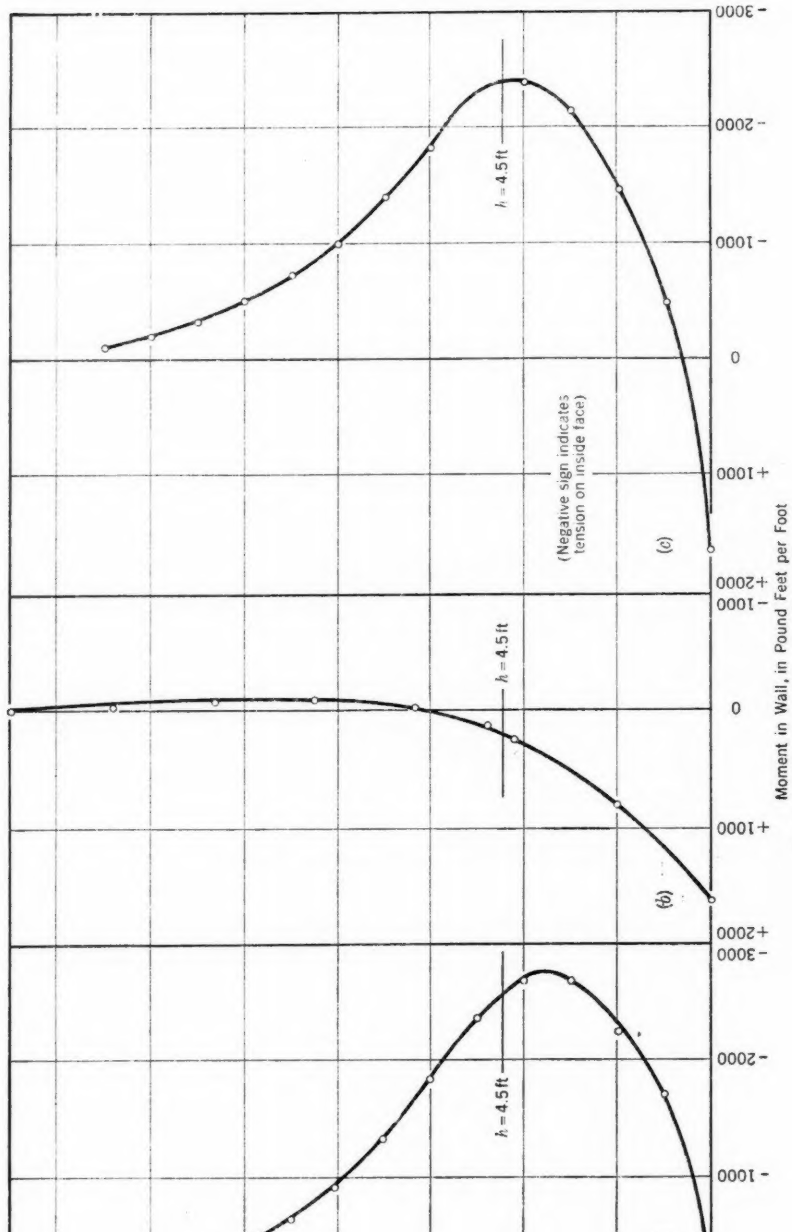

Fig. 2.—Moments in the Wall Due to:

(*a*) Horizontal Prestressed Wire
(*b*) Eccentric Loading of Vertical Steel
(*c*) Horizontal Prestressed Wire and Eccentric Loading of Vertical Steel

Station. Soon after completion, horizontal cracks developed in the walls. Leakage was substantial, and attempts to control it by patching and grouting the concrete wall were unsuccessful. After several months it was found, by examining the nuts on the top of the rods, that the stress of from 35,000 lb per sq in. to 40,000 lb per sq in. which had originally been imparted to the rods had virtually disappeared. The rods were retightened; and leakage was eliminated, only to reappear after a short interval, indicating that the rods were again loose. Results were similar when slip rods were used in other tanks. Large cracks, concentrated in a few locations, invariably appeared.

In later tanks smaller rods of intermediate grade were regularly used. Vertical slots were left in the concrete of the walls into which the rods were placed with anchors at the ends. They were then stressed to about 70,000 lb per sq in. after which the slots were filled with gunite thus bonding the rods into the wall concrete. For a time this seemed to be good practice. Later however horizontal cracks appeared in some of the larger tanks indicating that shrinkage and plastic flow were eliminating the prestress of the rods to a degree that it was no longer effective.

As a result it became evident that vertical shrinkage and flow in the concrete of tank walls are more dangerous than those taking place circumferentially and that at least an equal amount of overstress must be included to maintain continued resistance to wall moments. The use of the high-strength, cold-drawn wire was again indicated but from a practical standpoint it was not feasible until recently when machines were developed which would economically place it to the desired stresses. With the aid of these machines the vertical wire reinforcement is now regularly used at stresses slightly under the yield point and in quantity to provide ample additional unit compressive stresses in the walls, not only to take care of shrinkage and flow losses, but also to guard against temperature stresses which are incurred by seasonable weather changes.

The prestressing of the vertical wires alters, somewhat, the moment conditions that will exist after the band reinforcement has been placed. In Fig. 2(*c*), the combination curve shows that the point of maximum moment is at an approximate elevation of 4.5 ft. At this elevation, the moment induced by the band reinforcement is 2,600 ft-lb and that due to eccentric loading of the vertical wires is 200 ft-lb (Fig. 2(*b*)).

Using the conventional formula for the determination of the sectional area of the vertical reinforcement: $M = f_s A_s j d$; or $A_s = \dfrac{2{,}600 \times 12}{145{,}000 \times 0.9 \times 6.8}$ $= 0.035$ sq in. In this example, $d = 6.8$ in. instead of 5.5 in. as in the unstressed examples, and f_s is 145,000 lb per sq in. The difference in depth explains the reduced area A_s.

The wall of the empty tank has no vertical stress from shrinkage but will have compressive and tensile stresses due to the:

(*a*) Weight of dome and wall;

(*b*) Direct compression caused by the stressing of vertical wires;

(*c*) Stresses in the inside and outside faces due to eccentricity of prestressed vertical wires;

(*d*) Radial load of band reinforcement;

(*e*) Radial forces of shrinkage in concrete (partly eliminated by sliding joint); and

(*f*) Stresses of opposite sign in outside and inside faces due to temperature differences.

For this type of vertical reinforcement the summary in Table 3(*a*) shows an excellent condition with regard to shrinkage and flow. Both faces of the wall have a substantial surplus of compressive stress. However stresses due to variations in temperature must be added. Experience dictates that the sectional areas of the vertical wires must be considerably increased to provide insurance compression in the walls to prevent leakage even though, through some circumstance during construction, a horizontal fracture or weak plane has been left in the walls. Until the development of the machines for the vertical prestressing, the cost of the extra reinforcement was such that the engineer, unless he was very sure that it was needed, hesitated to require it in specification. The extra amounts required might vary with the locality but in general it would seem good practice to maintain a sufficient amount of residual stress horizontally across the wall section to exceed the unit pressure caused by the liquid by a large factor of safety.

The values shown in Table 3(*a*) are for the conditions that will exist at the instant that wire winding is completed. Subsequent losses of stress due to shrinkage and plastic flow will be substantial. After 4 or 5 years, however, most of the shrinkage and flow will have taken place. Table 3(*b*) has been prepared to cover this condition, and shows the vertical stresses remaining after all anticipated losses due to shrinkage and flow. The loss of stress in the vertical wires is assumed to be 35,000 lb per sq in., and thus the stress remaining in these wires is 145,000 lb per sq in. The respective columns in Table 3(*b*) show the approximate stresses in the concrete resulting from the vertical wire prestressing and the vertical and circumferential loads.

Despite the large amount of shrinkage and flow assumed to have taken place, there is little change in the vertical stresses from those shown in Table 3(*a*). The tank walls are stable and will not crack as has been repeatedly demonstrated by tanks reinforced in this manner.

The Construction of Prestressed Circular Structures

Experience gained in the design and construction of more than 500 tanks and other large annular structures leads to the following observations.

Concrete.—For best results, the concrete throughout the tank must be dense and free from voids and pockets. This can be accomplished through careful workmanship. An absorption factor less than 8% or 10% should prevent the concrete from disintegrating because of freezing, thawing, or general weathering.

Pneumatic Mortar or Gunite.—An encasement of pneumatic mortar has been proved to be the best protection for high-carbon wire after it has been stressed. Steel wire and rods coated with pneumatic mortar have been free from rust after exposure to tidal water for more than 25 years. There is no case on record of rusting having taken place where steel has been covered with a coating of good pneumatic mortar half an inch or more in thickness. This

is confirmed by studies on the corrosion of steel covered with pneumatic mortar conducted by the University of Toronto (Toronto, Ont., Canada) and reported in 1926.[17]

The bond between pneumatic mortar and wire is excellent. It can be readily demonstrated that a through cut in any of the wires encased in pneumatic mortar will result in negligible loss of stress. The increased diameter of the ends of the bonded wire, due to the release of stress, will prevent such loss except at the point of rupture. Should more than a $\frac{3}{4}$-in. thickness of pneu-

TABLE 3.—DESIGN OF VERTICAL REINFORCEMENT (STRESSES IN POUNDS PER SQUARE INCH)

Stress	Description	(a) BEGINNING OF PERIOD[a]			(b) AFTER FOUR OR FIVE YEARS		
		Computation	Inside face	Outside face	Computation	Inside face	Outside face
(1)	(2)	(3)	(4)	(5)	(3)	(4)	(5)
(a)	Weight of dome and wall......	$13+(h\times1.0)$	+ 30	+ 30	....	+ 30	+ 30
(b)	Direct compression, vertical wires; $f_c=\frac{A_s f_s}{A_c}$.............	$\frac{0.035\times180{,}000}{84}$	+ 75	+ 75	$\frac{0.035\times145{,}000}{84}$	+ 61	+ 61
(c)	Eccentricity, prestressed wires; $f_c=\frac{6\,M}{b\,d^2}$..................	$\frac{6\times200}{1\times7\times7}$	+ 25	− 25	$\times\frac{145{,}000}{180{,}000}$	+ 20	− 20
(d)	Radial load, band reinforcement; $f_c=\frac{6\,M}{b\,d^2}$..................	$\frac{6\times2{,}600}{1\times7\times7}$	−318	+318	$\times\frac{100{,}000}{140{,}000}$	−227	+227
(e)	Radial forces of shrinkage......	....	....	....	....	....	....
...	Empty tank; total stress (a) to (e)......................	....	−188	+398	....	−116	+298
...	Add full tank, liquid load; $f=\frac{6\,M}{b\,d^2}$	$\frac{6}{49}\times2{,}600\times\frac{100{,}000}{140{,}000}$	+227	−227	....[b]	+227[b]	−227[b]
...	Summation of stresses, full tank	....	+ 39[c]	+171[c]	....	+111	+ 71

[a] The instant that wire winding is completed. [b] Same as for stress (d) but with opposite sign. [c] Assuming no plastic losses.

matic mortar be applied, a layer of light wire mesh should be included to prevent sagging and reactions caused by temperature differences.

High-Carbon Cold-Drawn Wire.—The high-carbon, cold-drawn wire used for the horizontal and vertical prestressing of tank walls is one of the most reliable of all building materials because of the repeated working and reworking under high stresses to which it is subjected during manufacture. The wire is packaged in coils at the factory and delivered to the site wrapped in water-repellent paper to prevent rusting prior to use. During the wire-winding operation, each coil is jointed to the next coil by special, patented, spring-loaded splices. Tests made under the supervision of the writer demonstrate that less than 5% of the stress originally imparted to the wire will be lost from

[17] "Gunite as a Protection for Steel Structures," by Peter Gillespie and P. J. Culleton, *Bulletin No. 4*, Univ. of Toronto, Toronto, Ont., Canada, 1926, p. 109.

creep or plastic flow of the wire through the passage of time. Fig. 3 illustrates a self-propelled machine that winds wire around a tank at the uniform unit stress of the design, thus placing the walls in permanent circumferential compression. The operators on the ground are applying a pneumatic mortar coat that bonds the wire to the structure and protects it against corrosion.

Floors.—It is of the utmost importance that the foundation for the structure be stable, especially where the joint between the floor and the side wall is free. Until recently dowels have been included to tie floor and side walls rigidly together. Where this practice has been followed, there have been no difficulties

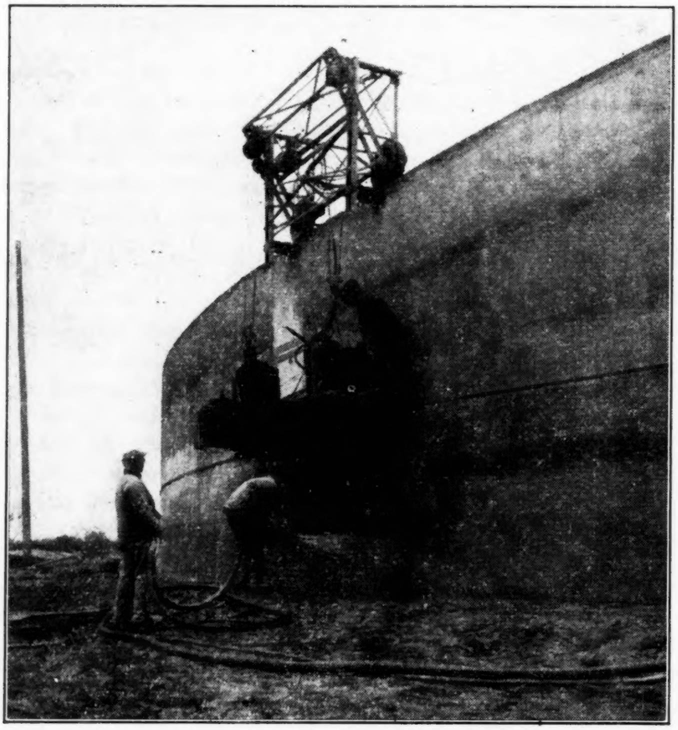

FIG. 3.—SELF-PROPELLED MACHINE THAT WINDS WIRE AROUND A TANK AT A UNIFORM STRESS

from leakage at the joint. Where the side wall is tied to the floor, the wall serves as a deep beam and supports the edge of the floor against settlement, although the edge of the floor must be relatively thick to take the increased shear stress. In general, a fixed joint is used with tanks having diameters less than 100 ft. If free joints are desired, however, they can be made bottle tight without too much difficulty.

Experience indicates that relatively thin floors with a high percentage of reinforcing steel will be the most watertight and permanent. If pneumatic mortar is used, a floor approximately 2 in. thick, reinforced with wire mesh having a sectional area of 0.5%, will conform best to earth movements with

less leakage. It will also prevent the passage of any free water under heads of 100 ft and more. This is in accordance with the history of such floors and may be demonstrated by the use of hydraulic testing machines.

Where a poured concrete floor is used, a 4-in. thickness with a $\frac{5}{8}$-in. topping of pneumatic mortar is satisfactory if the sectional area of the reinforcement in each direction is equal to 0.5% of that of the concrete. The pour, however, should be continuous so that construction joints are eliminated. The topping of pneumatic mortar, which should not be applied until just before the tank is to be filled, will effectively seal any shrinkage cracks that may occur during the initial curing.

After the floor has been poured or shot, it must be kept saturated with water until the walls have been constructed to prevent it from shrinking until that time. By using this procedure the walls and the floor will shrink together to some extent, and thus prevent excessive moment in the walls. This procedure is mandatory when floors and walls are doweled together.

Thicker concrete floors with smaller percentages of reinforcement cannot be expected to bend or conform to earth movements without cracking and leaking. In addition, they require expansion joints, which almost always leak.

Walls.—Sufficient surplus stress to offset maximum shrinkage and plastic flow of concrete must be imposed on both horizontal and vertical wall reinforcement. The minimum surplus allowance for successful circumferential prestressing should not be less than 40,000 lb per sq in. for rod reinforcement or 35,000 lb per sq in. for high-carbon, cold-drawn wire. The apparatus for circumferential prestressing with wire (see Fig. 3) is a self-propelled traction machine, hung from the top of the tank, which travels continuously around the walls.[18] A sprocket operated by a gasoline engine engages an endless linked steel belt which is passed about the perimeter of the structure. A spring-loaded take-up maintains the tension necessary for adequate traction. An automatic power drive controls the vertical movement of the machine insuring accurate spacing of the wire (Fig. 4), which is taken from a reel generally mounted on the working platform of the machine.

The desired stress is imparted to the wire by drawing it through an aperture of smaller diameter. For the convenience of the operator, a dial gage on the machine registers the load being imparted to the wire. Wire winding may start at the bottom or the top of the wall from an anchor set in the concrete. It takes only about 2 days to wind the walls of an average million-gallon water tank. With larger tanks, the walls may require several layers of wire; if so there is no added difficulty. Each layer is covered with a minimum thickness of pneumatic mortar and then the succeeding layer is applied.

In view of the large number of wires required for the vertical prestressing of large tanks it has been necessary to develop special techniques and equipment to place and stress the wire in groups of six or more. For large tanks special anchors are placed in the concrete at the top and bottom of vertical keys on the inside face of the wall. An assembly of wires is then affixed to the top anchor, and is stressed at the floor level of the tank by a machine that

[18] "High Stressed Wire in Concrete Tanks," by J. M. Crom, *Engineering News-Record*, Vol. 131, 1943, p. 947.

stretches the strands simultaneously and forces "home" a coupling pin to the bottom anchor when the wires have reached the proper stress. Sometimes, the wires are anchored in the keys at the bottom of the wall of small tanks and are stressed individually from the top of the wall by hydraulic jacks. As each wire is brought to the desired stress, a special anchorage device is affixed to the wire to hold it in the stressed condition. In both large and small tanks, after

Fig. 4.—Spacing and Covering of Prestressed Wire

the wire is fully stressed, the keys are filled with pneumatic mortar to bond the prestressed wire into the main body of the structure, and to protect the wire permanently from corrosion. Recently in order to provide more reinforcement for very thick walls, cable assemblies with cast anchor plates at the top of the wall were used. These cables are assembled as "hairpins" and are sheathed to provide the slippage which occurs during the stressing operation. The units are placed in the walls during construction.

Domes.—It is not within the scope of this paper to discuss the theory and design of prestressed domes, which have been thoroughly examined by M. F.

Fornerod,[19] M. ASCE. It is sufficient to state that for large domes, with proportionately high band loads, multiple layers of high-carbon wire may be concentrated economically and efficiently around the points of maximum reaction by the previously described machines. The cost of such domes is substantially below that of any other type of permanent covering, including rectangular truss and precast roof construction. These domes, which are only a few inches in thickness and eliminate the need for supporting columns, are fireproof and need little maintenance.

Although dome roofs of this type were developed principally in connection with tanks, they have been adapted to other structures such as sports arenas, aircraft hangars, shopping centers, and motion picture theaters where an unobstructed roof surface which permits good natural or artificial light diffusion was desired.

Summary and Conclusions

One of the principal purposes of this paper has been to discuss and analyze the results of tests conducted by the Massachusetts Institute of Technology to measure the shrinkage and plastic flow of concrete under stress, and to call attention to their importance in connection with the design of prestressed concrete structures. These tests have proved that shrinkage and flow can be sufficiently great to nullify most of the prestress which designers formerly attempted to impart to the wall reinforcement of tanks and other circular structures. The failure of many tanks is thus accounted for.

On the basis of these tests and other tests it is disclosed that, using unit steel stresses, developed without friction, considerably in excess of 100,000 lb per sq in., a balance of prestress, ample for working purposes, can be maintained in the reinforcement after all losses due to shrinkage, plastic flow, and other factors have taken place.

In the case of tanks and other large circular structures the required steel stresses for the walls can be attained through the use of machines that simultaneously place and stress without friction high-carbon, cold-drawn wire to the desired specifications, thus making possible the economical construction of tanks of very large diameter and capacity, and, when desired, of large-span, thin-shelled dome covers, unsupported except at the rings.

Most of the designed dimensions for prestressed circular structures are based on actual tests and simple, direct computations that do not require the complicated assumptions necessary for conventional reinforced concrete.

This paper also presents certain observations from experience on the construction of prestressed circular structures. When all factors are considered, it seems conclusive that large annular concrete structures, reinforced with highly prestressed wire are more rational and safer than those built of conventionally-designed reinforced concrete. Fortunately, because of a saving in labor and materials, together with low first cost and maintenance, prestressed tanks are also more economical.

[19] "Prestressed Concrete Shell Roof Construction," by M. F. Fornerod, *Publications*, International Assn. for Bridge and Structural Eng., Vol. 8, 1947, pp. 91–103.

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